Before the Independent Hearings Panel
at Waimakariri District Council

under: the Resource Management Act 1991

in the matter of: Proposed private plan change RCP31 to the Operative Waimakariri District Plan

and: **Rolleston Industrial Developments Limited**
Applicant

Statement of Evidence of Eoghan O'Neill

Dated: 6 July 2023

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EVIDENCE OF EOGHAN O'NEILL

1 My full name is Eoghan Michael O'Neill.

2 I am a Technical Director with Pattle Delamore Partners Ltd and have been employed in that capacity since October 2012. I am a Chartered Professional Engineer with approximately 20 years' experience in the planning and design of wastewater, water supply and stormwater infrastructure.

3 I hold Bachelor of Engineering and Master of Engineering Science degrees awarded by University College Dublin. Much of my experience is related to the planning of infrastructure to facilitate development. I have prepared and presented evidence to Plan Change Hearings, Resource Consent Hearings and the Environment Court on numerous occasions. I have performed this role both as a Council employee and as a consultant on behalf of applicants.

4 I am familiar with the plan change application by Rolleston Industrial Developments Limited (the *Applicant*) to rezone approximately 156 hectares of land 535 Mill Road, Ōhoka to enable residential development and two small commercial zones.

CODE OF CONDUCT

5 Although this is not an Environment Court hearing, I note that in preparing my evidence I have reviewed the Code of Conduct for Expert Witnesses contained in Part 9 of the Environment Court Practice Note 2023. I have complied with it in preparing my evidence. I confirm that the issues addressed in this statement of evidence are within my area of expertise, except where relying on the opinion or evidence of other witnesses. I have not omitted to consider material facts known to me that might alter or detract from the opinions expressed.

SCOPE OF EVIDENCE

6 My evidence will deal with the following:

6.1 Description of the management of stormwater within the proposed development;

6.2 Assessment of options available for servicing the site for wastewater;

6.3 Response to s42a Officers Report; and

6.4 Response to other concerns raised by submitters related to stormwater management or wastewaters servicing.

7 In preparing my evidence, I have reviewed and considered the following:

7.1 The updated Outline Development Plan (*ODP*);

7.2 Section 42a Report on Private Plan Change Application 31 prepared by Mr Andrew Willis on behalf of Waimakariri District Council (*WDC*);

7.3 Statement of Evidence (Infrastructure) prepared by **Mr. Tim McLeod** of Inovo Ltd;

7.4 Statement of Evidence (Hydrology) prepared by **Mr. Bas Veendrick** of Pattle Delamore Partners Ltd;

7.5 Statement of Evidence (Ecology) prepared by **Ms. Laura Drummond** of Pattle Delamore Partners Ltd; and

7.6 Statement of Evidence (Flooding) prepared by **Mr. Ben Throssell** of Pattle Delamore Partners Ltd.

SUMMARY OF EVIDENCE

8 The management of stormwater quantity, including hydraulic continuity between the upstream and downstream catchments, can be managed by means of the following:

8.1 Formalised flow paths to be installed to connect the upstream and downstream catchments.

8.2 Attenuation and flood storage to be provided within the site to manage runoff up to the 2% AEP (50-yr ARI) by the use of basins, compensatory storage, and rain tanks. Stormwater detention basins will be designed to be constructed along the fall of the site with minimal excavation to ensure storage can be provided without intercepting highest groundwater at the site. Low bunding shall be gradually formed along the fall of the contour to retain floodwaters.

9 Water quality treatment can be provided as follows:

9.1 Residential and retirement village/school runoff to be predominantly treated by means of filtration via high infiltration rate raingardens or swales and bioscapes which will be designed to treat 90% of rainfall runoff from the site. Raingardens and bioscapes, being approximately 1m deep, will likely be constructed into seasonal groundwater. They will be fully lined so as to avoid any active drainage of groundwater that may be intercepted at their base.

9.2 Up to 2ha of stormwater wetlands or wet ponds can be constructed at the site as a permitted activity under Rule 5.114 of the Canterbury Land and Water Regional Plan (*LWRP*). This provision allows greater flexibility for the location of potential treatment and storage facilities in wetter parts of the site during detailed design. For the purposes of this Plan Change concept, all storage and treatment is provided without the use of wetlands or wet ponds.

9.3 Large lot residential stormwater runoff to be treated by means of swales, high-infiltration raingardens and bioscapes.

9.4 Stormwater runoff from business areas to be treated by means of rain gardens or proprietary filtration devices.

9.5 All stormwater treatment infrastructure will be designed to limit potential groundwater take to within permitted activity status under requirements of the LRWP.

10 Wastewater for the proposed development can be managed by way of a new wastewater pump station located within the plan change area pumping to Rangiora WWTP via a new rising main.

11 To facilitate the initial build out of lots, and mitigate any odour issues which would occur with a small number of lots connected to the new wastewater main, the new pump station could connect to the existing Mandeville/Ōhoka wastewater pressure main to facilitate the development of an initial 250 lots before the new pressure main was constructed to the WWTP.

EVIDENCE

Plan Change Summary

12 The majority of the PC31 site is located at 535 Mill Road and is roughly trapezoidal in shape bounded for the most part by Whites, Mill and Bradleys roads, Ōhoka. The site is typically gently sloping (1:180) to flat, sloping from west to east towards Whites Road. The current land use of the plan change site is a dairy farm with the farmhouse and farm buildings in a cluster towards the western corner and an additional cluster of farm buildings near the boundary of 531 Mill Road. Open paddocks predominate, but the site comprises a variety of mature trees and shelterbelts. A relatively high water table extends over the site and several waterways, including Ōhoka Stream and the Ōhoka South Branch, flow in an easterly direction across the site.

13 The proposed residential development will comprise of up to 850 residential units, a potential primary school and a potential retirement village. If a school is not developed, approximately 42 additional residential units could be developed. The two new commercial areas

(Business 4 Zone) will provide for approximately 2700m^2 of commercial floor space and car parking.

Existing Site Stormwater Characteristics

14 The proposed site is zoned as rural and is approximately 156 ha in area. The existing land-uses on-site consist of large undeveloped paddock areas. Existing impervious areas are limited to unsealed roads, buildings and associated sealed areas (< 1% of existing area). The general fall across the site is northwest to southeast and elevation ranging between RL 29 m to RL 20 m. The average slope across the site is approximately 0.5% (1V:200H).

15 The site has limited stormwater infrastructure and runoff from the site generally drains via land drains or as sheet flow from the site. These existing land drains collect and drain stormwater and high groundwater away from the contributing catchment areas to the main waterways crossing the site. As shown in **Attachment 1**, a tributary of Ōhoka Stream crosses the northern part of the site, and several branches of South Ōhoka Stream cross the southern half of the site. Two springs are mapped on the site in the Canterbury Regional Council (ECan) online database. A groundwater seep is located on the site closer to Whites Rd. These springs are discussed in detail in the evidence of **Ms Laura Drummond**.

16 Potential flooding of the site and the surrounding land is covered in detail in the evidence of **Mr Ben Throssell**. The stormwater management proposals for the plan change area have been developed in close collaboration with the flood modelling and flood mitigation work to ensure that the development can progress without increasing the flood risk to properties upstream or downstream of the development. As noted in Paragraph 114 of **Mr Ben Throssell's** evidence, *"modelling of the 200-year event shows the flood hazard is still low for areas south of Mill Road/downstream of Whites Road and moderate for areas north of Mill Road. I note the PDP model predicts generally no effect greater than 10 mm for areas north of Mill Road and no increase greater than 20 mm for habitable dwellings elsewhere within the PDP model."* He therefore concludes that the effect of the development on flooding outside of the plan change area are less than minor.

17 In general, the groundwater flows to the southeast, towards the coast. Groundwater discharges into spring fed streams, including the Ōhoka Stream and the Cam River/Ruataniwha. The groundwater is typically shallow and subject to seasonal fluctuations. Groundwater at the site is estimated, using the record from bore M35/0596, to be an average of 0.64 m below ground level (bgl) with the highest recorded groundwater level at 0.14 m bgl (June 2018). Seasonal fluctuations in this bore are relatively small, commonly being 0.5 – 0.8 m. As expected, groundwater

levels are generally highest in winter/spring and lowest in summer/autumn. It is noted that bore M35/0596 is close to spring M35/7485 (mapped location is 20 m away), and so may be in an area of the site that has particularly high groundwater levels.

18 It is noteworthy that extensive test pitting undertaken by Tetra Coffey Ltd at the site in May 2021 encountered a range of groundwater depths, these are shown on **Attachment 1**. The shallowest groundwater level recorded during this testing was 1.15m below ground level close to Spring M35/7485, the deepest groundwater was encountered at 1.85m below ground level at the Mill Rd end of the site. The recorded water depth at monitoring bore M35/0596 at the time of these investigations was approximately 0.9m below ground level. Detailed knowledge of maximum ground levels across the site will be crucial to inform the placement and depth of stormwater detention ponds at the site. The stormwater concept has conservatively assumed that stormwater detention basins will be constructed with minimal excavation (less than 0.2 m) to avoid interception of groundwater. Detailed groundwater monitoring at the site will be undertaken prior to development to inform the detailed design of these basins and ensure no interception of groundwater occurs.

19 The downstream catchment has comparable properties to the pre-development site. The downstream catchment is undeveloped rural land (paddocks) with land drains collecting runoff and intersecting shallow groundwater. The downstream catchment drains towards the Ōhoka Stream and eventually to the Kaiapoi River.

Flow Continuity

20 The continuity of pre-and post-development flows from upstream of the site to downstream of the site will be provided by way of the three main formalised flow path corridors through the proposed development. The management of these flow corridors to convey flow from upstream of the site to downstream of the site without increasing the flood risk outside of the plan change area is discussed in in the evidence of **Mr Ben Throssell**.

Proposed Stormwater Management

21 The pre- and post-development stormwater catchments for the proposed development were delineated using the following information:

21.1 Available LiDAR information;

21.2 Existing stormwater infrastructure as per the WDC GIS;

21.3 Flow paths as determined by the WDC 200-year flood modelling results; and

21.4 Proposed development plans.

22 The following design criteria was used as the basis assessment of the stormwater effects:

Table 1: Design Criteria for Stormwater Treatment Devices

Item	**Design Criteria**	**References**
Primary flows	• 5-year return event (20% AEP)	WDC CoP 2020
Secondary flows	• 50-year return event (2% AEP)	WDC CoP 2020
Attenuation requirement	• Post-development peak flows for all intensities to be less than pre-development flows	WDC CoP 2020
Rainfall	• HIRDS V4 RCP 8.5 (2081 – 2100)	WDC CoP 2020
Runoff coefficient	• As per Table 5.2 & Table 5.3 of WDC Engineering Code of Practice	WDC CoP 2020
Water Quality Flow	• 5 mm/hr	CCC Onsite Stormwater Mitigation Guide

23 Any modification of the main flow paths (i.e. Ōhoka Stream tributaries) across the site will be designed to maintain hydraulic connectivity between the upstream and downstream catchments including baseflow. They also will collect and convey controlled outflows of treated stormwater, from the attenuation basins associated with the proposed development catchments, towards the downstream environment. All stormwater treatment infrastructure and stormwater attenuation basins will be located outside of the 50-year flood level of the main flow paths.

24 Primary stormwater runoff from the residential development areas (i.e. flows from up to a 20% AEP/5-yr ARI) within the site will be collected along roads via swales. This flow will be conveyed to be discharged into either raingardens or bioscapes for treatment.

25 Rain gardens are a "closer to source" treatment system consisting of engineered gardens designed to harness the natural ability of

vegetation and soils to treat stormwater. They are typically built in the berm to the side of the kerb and channel, or in this instance would be in the swale or the berm next to the swale. Treatment occurs through sedimentation, filtration, adsorption through the soil media and uptake by vegetation. A proprietary high infiltration rate engineered media (trade name Filterra®) can also be used within rain gardens to reduce the required footprint or increase the treatment flow over a certain raingarden infiltration area. Raingarden media will absorb and filter contaminants before stormwater flows into a slotted or perforated pipe underdrainage system located within a granular drainage layer at the base of the rain garden structure. The raingardens will be located within a concrete structure or appropriately lined excavation to prevent the drainage of groundwater into the system. The underdrainage system connects to a piped stormwater network which conveys treated stormwater along with any first flush overflow to detention storage. A typical cross section though a rain garden is shown in Figure 1 below. Note the figure below allows for partial soakage to ground, whereas the systems within the Plan Change area will be fully sealed.

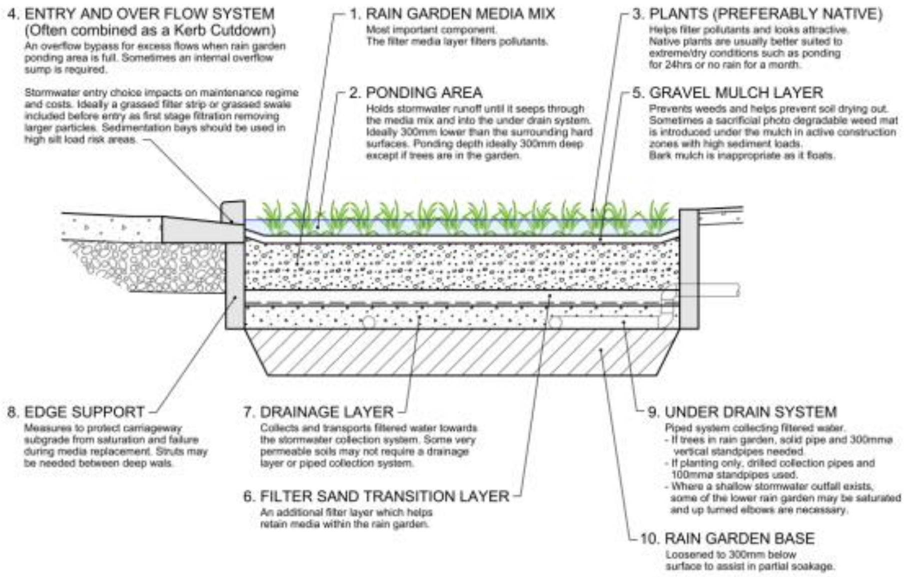


Figure 1 – Typical Rain Garden Cross- Section (Source: CCC Rain Garden design, construction and maintenance manual)

26 Bioscapes are effectively a larger form of rain garden which can be located to receive larger cumulative flows from a catchment or sub-catchment. Bioscapes are proposed to be constructed using Filterra® engineered media. As with raingardens, the treated stormwater along with any by-pass flow will be collected by underdrains to a piped stormwater network and conveyed to detention facilities.

27 As with raingardens, bioscapes will be fully lined and no groundwater will be able to enter the system, therefore no groundwater take consent will be triggered. Depending on the time of year, the construction of both rain gardens and bioscapes have the potential to intercept groundwater during construction. This can be managed by undertaking construction in the summer months or via temporary construction dewatering consents for the development which will also be required for pipeline construction and other activities.

28 Treated stormwater and bypass stormwater in excess of the Water Quality flow will enter the piped stormwater network which shall be designed to convey primary flows up to the 20%AEP. Flows in excess of the primary flows (i.e. flows in excess of a 20%AEP) will be directed towards the detention basins via roads and dedicated easements/swales. The Business areas will have conveyance pipelines sized for the 10% AEP with treatment via raingardens, bioscapes or proprietary filter devices in cark parks or other green space. Flows will then be conveyed to detention areas.

29 Attenuation is to be provided across the development for management of the post-development discharge to the waterways to ensure that this does not exceed the pre-development runoff from the development. Formalised attenuation will be provided for up to the 2% AEP event by means of attenuation basins located at the end of the stormwater pipe network for sub-catchments. These basins will have controlled outlets discharging into the main flow path corridors. The total peak discharge flow from these outlets will be equal to or less than the peak pre-development. The balance volume will be stored within the detention ponds and released over an extended period of time as the storm recedes. Attenuation tanks are also proposed to capture and attenuate roof runoff for rural-residential areas, however the benefit provided by these storage volumes have not been considered as part of the detention basin volume calculations.

30 The required stormwater attenuation for each catchment up to the 2% AEP (50-year ARI) has been calculated. The volume is based on matching (post-development) the pre-development runoff for the site during the critical storm duration and taking into account the change in land use. The pre-development peak runoff for each catchment has been calculated using the Rational Method along with appropriate runoff coefficients and time of concentration for the pre-development situation. This is then applied to a calculation of a hydrograph for catchment runoff for each event duration. The pre-developed hydrograph was created using the Standard Rational method. The post-developed hydrograph was developed using the Modified Rational method and using the peak flow of the pre-developed situation. After that, a detention pond for each catchment

was sized using the pre-developed peak flow as the target detention pond outflow. The post-developed Modified Rational hydrograph was the pond's inflow hydrograph. This is considered to be a very conservative approach as the pre-development flow calculated is low relative to the equivalent pre-development flow calculated by an alternative methodology using the WDC District flood model hydrology. As the pre-development flow is used to set the controlled outlet flow from the detention basins, a higher pre-development flow would result in a lower volume of storage.

31 The total attenuation required for the development using the Rational Method Hydrograph approach is 21,990 m^3, this will be provided within a number of stormwater detention basins across the development totalling approximately 52,195 m^2 of basin area. As a comparison, the total detention volume calculated by an alternative methodology using the WDC District Flood Model hydrology is approximately 10,000 m^3. Stormwater detention basins are proposed to be constructed outside of the 2% AEP (50-year ARI) flood level of the flow path corridors. Indicative detention pond locations are indicated in **Attachment 2** of my evidence.

32 Given the uncertainty regarding highest groundwater depths across the site as discussed in Paragraphs 17 to 19 of my evidence above, the detention basins have been conceptually designed so as to require no more than 200mm depth of excavation. Stormwater detention basins will be designed to be constructed along the fall of the site with minimal excavation being undertaken to ensure that storage can be provided without intercepting highest groundwater at the site. The site is generally well graded in a west to east direction at a fall of approximately 1 in 200. Low bunding shall be gradually formed along the fall of the topography to retain floodwaters within the basins. Maximum bund height at the downgradient end of the proposed basins shall be approximately 1m in height. The maximum pond water depth shall therefore vary from 0m at the upgradient end of the basin to a maximum water depth of 0.8m at the downgradient end of the basin. Maximum depth shall be controlled by a backwater overflow at the upgradient end of the basin which will direct sheet flow to the main flow corridor on the outlet site of the basins. These shall be incorporated into the landscape with appropriate planting treatments. Each basin shall have a controlled outlet, with the total outlet flow from all basins to be no more than the calculated pre-development flow.

33 Some small areas of development at the Whites Rd end of the site will likely be difficult to attenuate in the above manner. In such an event, it is proposed that the treated flow from these areas will discharge directly to the main flow path corridors with no attenuation, but this will ideally be kept to a minimum. To ensure hydraulic neutrality is maintained between pre- and post-

development flows, additional compensatory storage will be provided within other stormwater detention basins within the development, along with a reduced basin outflow at those locations, to compensate for those areas that will not be attenuated. The overall impact will be neutrality between pre-development and post-development flows.

34 In the event that groundwater levels are determined to be deeper in certain parts of the site during detailed groundwater monitoring, the detention basins can be excavated slightly deeper into the ground. However, the above concept provides a robust and conservative solution that can adequately manage stormwater on the site without the need for basin which actively intercept and take groundwater.

35 As a general rule, stormwater treatment areas will be located an appropriate distance away from springs and streams. Untreated stormwater will be managed such that it cannot come into contact with a spring or discharge into a stream. Stormwater detention areas, which will receive treated stormwater and first flush bypass flow will be located away from springs and above the 50-year flood level.

36 It should be noted that up to 2ha of stormwater wetlands or wet ponds can be constructed at the site as a permitted activity under Rule 5.114 of the LWRP. This provision allows additional flexibility for the location of potential treatment and storage facilities in wetter parts of the site during detailed design. For the purposes of this Plan Change concept, all storage and treatment are provided without the use of wetlands or wet ponds.

Proposed Wastewater Servicing

37 The proposed plan change site is not currently serviced for wastewater. The site is located between Mandeville to the south and Ōhoka to the north. These areas are serviced by the existing Mandeville Ōhoka wastewater scheme. This consists of two sub catchments. The main catchment, Mandeville, consists of a network of Septic Tank Effluent Pumping (*STEP*) systems which discharge to a central pump station on Bradleys Rd. From here primary effluent is pumped through a pressure main to the Rangiora wastewater treatment plant (*WWTP*). Wastewater from the Ōhoka catchment is collected via a pumped sewer network which connects directly into the Mandeville pressure main as it passes through Ōhoka.

38 It is my understanding from **Mr Tim McLeod's** evidence that the servicing of the Plan Change area for wastewater is proposed to be either via conventional gravity or pressure sewer to a new pump station near Mill Rd. I concur with **Mr Tim McLeod's** evidence that a pressure sewer network at the site would be preferable. A

pressure sewer network is such that each property has a single pump station with progressive cavity grinder pumps. These pump into a network of welded polyethylene pipes of relatively small diameters which would provide far less opportunity for ingress of groundwater as opposed to a conventional gravity sewer system across the site. The pressure sewer network would connect to a main pump station within the Plan Change area which would in turn pump to the WWTP. In an earthquake situation, pressure sewer networks are known to be significantly more resilient and far easier to repair than conventional gravity pipes. In Christchurch they are being used extensively in greenfield areas, particularly South East Halswell, where ground water levels are similarly high.

39 The proposed new pump station for the Plan Change area is to connect via a new separate pressure main to the WWTP. Assuming that the existing pipeline will follow the general route of the existing pressure main, the proposed pipeline would be approximately 7.1 Km long. It would initially have a falling grade to the Cust Drain and would then rise again to the WWTP site. There are a number of obstacles along the route which would need to be considered in design. These include three crossings of Ōhoka Stream Tributaries, a crossing of the Cust Main Drain, a railway crossing at Lineside Rd and the ultimate connection to the WWTP. All of these obstacles have been overcome previously in the design and construction of the existing pipeline and, in my view, similar design solutions would be equally successful for the proposed pipeline and therefore show this is a viable option.

40 In his evidence, **Mr Tim McLeod** raises the possibility of initially connecting the proposed development into the existing pressure main from Bradleys Road pump station until such a time as a reasonable number of lots within the plan change area have developed. I would concur that there are advantages to this approach. Managing the initial build out of flows from new lots through a new long pipeline would present challenges with respect to odour management at the discharge location and any air valve locations along the pipe route. Connecting the new pump station to the existing pipeline would provide a dual benefit of reducing the hydraulic retention time for the existing pipe situation, as well as allowing for a sufficient level of development to occur within the plan change area prior to connection of that flow to a new 7.1 Km long pipeline. I have carried out some independent analysis to determine if, in my view, there is capacity available in the existing pressure main to cater for this flow.

41 Analysis of historical Bradleys Road pumped wastewater flows from 2021 to 2023 (SCADA data at 1-minute intervals supplied by WDC) indicate that this pump station and pressure main are generally running significantly below the capacity of the system for the

majority of the time. The average daily flow over the period assessed was 269 m^3/day compared to a theoretical flow capacity of approximately 2340 m^3/day with a pump at Bradleys Road running continuously through the pressure main, this is just 11.5% of the theoretical flow capacity.

42 However, during certain conditions when groundwater levels in the area west of Mandeville are particularly high, the wastewater flows into the Bradleys Road pump station have been noted to increase dramatically. This was evident in the wastewater record for July/August 2022 and similar groundwater conditions are also noted to have occurred in June 2014.

43 Following a significant series of rainfall events in July 2022, the wastewater flows into Mandeville pumps station were significantly elevated for an extended period of time. Flows from the Bradleys Road pump station exceeded 1000 m^3/day for approximately 12 days, with the highest daily flow recorded on 31st July 2022 at 1740 m^3/day. On this peak day, the pump station was operational for more than 17 hours. Therefore, even in very extreme circumstances within the wastewater network, ultimately caused by historically high groundwater levels, there were still approximately 7 hours when the main Bradleys Road pump station was not operational.

44 It can therefore be concluded that, the existing Bradleys Road Pump Station pressure main does have some limited spare capacity under peak flow conditions to receive limited pumped flows from further development in Ōhoka. The utilisation of this capacity would require direct communication between Bradleys Road pump station and a new pump station at the proposed development. The control logic for both pump stations would need to be established such that both stations cannot pump at the same time and the new pump station would only operate when Bradleys Road pump station storage tank volume was below a certain tank level at the pump station. For the majority of time, there would be little to no restriction on the ability for each pump station to operate.

45 Following prolonged wet periods, such as occurred in June 2014 and July 2022, there is a smaller window of available capacity within the pipeline. Based on the peak day flow from 31st July 2022 there is approximately 6.5 hours of pumping time available into the pressure main. Taking a conservative approach, it is assumed that 4 hours of this would be available to the new Plan Change pump station. Assuming a pressure sewer network was established to service the development, the proposed new pump station would have a design flow of approximately 12.4 L/s. At 4 hours of pumping, this would equate to approximately 179 m^3 of available wastewater capacity in

the pipeline on a peak day. This equates to approximately 260 lots worth of pumping capacity.

46 In order to avail this capacity, sufficient buffering capacity would need to be available within the Plan Change area's wastewater system to store and buffer flows while the Bradleys Road pump station is operating. The proposed new pump station will have a standard requirement for emergency storage to be provided in order to allow a minimum response time for power failures or major mechanical issues. This requirement is typically 8 hours of storage at average flows, for the proposed Plan Change area pump station this equates to approximately 233 m^3 of storage which would need to be constructed at the same time as the pump station.

47 While this total emergency storage allowance would be available from the date of construction, the total storage provided would be significantly greater than the actual emergency storage required as the development commences and progresses. For example, at 100 lots of development the actual 8-hour emergency storage requirement is 23 m^3. As development progresses, the emergency storage requirement gradually increases to meet the total storage provided at the time of construction. At 250 lots worth of development completed, the emergency storage requirement is 57 m^3 of storage, leaving an additional 176 m^3 of storage potentially available to buffer flows at the pump station while Bradley Road pump station is operating. This available storage of 176 m^3 is more than 24 hours of total daily wastewater volume for 250 lots. This would be more than sufficient to buffer flows into Bradleys Road during peak periods of flow. I would therefore conclude that there is capacity within the Bradleys Road pressure main for a minimum of 250 lots from the Plan Change during peak periods of flow. Above 250 lots, a new pressure main to WWTP would need to be constructed.

48 The WWTP has been upgraded over the last 10 years to create significant additional capacity for growth within the district. Upgrades include the construction of a new Aeration Basin as well as the construction of a new inlet works structure which was designed to receive up to 960 L/s as an ultimate design flow. I understand that, at present, the inlet works have the capacity to receive approximately 750 L/s which can be increased to the ultimate design flow of 960 L/s via modifications to the inlet screens within the inlet structure. This ultimate flow of 960 L/s represents a population equivalent capacity of approximately 66,000. It is recognised that the existing treatment capacity of the WWTP does not match this ultimate design flow.

49 Further upgrades to the WWTP and the downstream disposal infrastructure are planned within the current Long Term Plan (*LTP*)

and have been earmarked for future LTP's. As Mr Roxburgh acknowledges in his evidence, *"Forecast population growth vs density of development are two subtly different things, and strategic assets like the Rangiora WWTP are designed with population growth rather than development density in mind"*. I agree with Mr Roxburgh that, at this trunk infrastructure level, upgrades are considered on a district wide growth basis. It is not anticipated that the approval of this Plan Change would have a significant impact on the district wide rate of growth within the Waimakariri District particularly given the development will be staged. Upgrades to the Rangiora WWTP, and the timing of such to cater for district wide growth, will be considered by WDC though the three yearly LTP review process and timed accordingly.

RESPONSE TO SECTION 42A REPORT

Stormwater Infrastructure

50 In his evidence, Mr Roxburgh notes that *"Groundwater will be the main challenge to ensuring viable stormwater infrastructure can be provided across the plan change area to ensure stormwater neutrality, and treatment of stormwater, is achieved as per the ECoP standard."* Paragraphs 21 to 36 of my evidence above describe in detail how the proposed stormwater system will collect, treat and manage stormwater at the site. Collection, treatment and disposal of stormwater will use the natural grade of the land across the site to achieve sufficient fall and head to convey flow through the stormwater collection and treatment systems to the detention areas. Detention concepts have been developed on a very conservative basis both for size and depth to ensure that groundwater interception will not be an issue.

51 Treatment systems will be sealed so as not to permit ingress of groundwater and will be designed such that treatment media would not operate under submerged conditions.

52 In Paragraph 43 of his evidence Mr Roxburgh notes *"PDP propose a wetland within the treatment train for WQ1 and WQ10".* Mr Roxburgh goes on to point out the consent issues associated with the use of wetlands in the original PDP report. Mr Willis also notes this as part of his evidence and his decision. The wetlands proposed in the initial PDP report were intended to be small and sized so as to stay within LWRP permitted activity rules for water takes. The concept has subsequently been revised so as to remove these wetlands but, as noted in my evidence, wetlands and wet ponds up to 2ha could be constructed as permitted activities within the site. At detailed design phase, with more detailed information regarding groundwater at the site, their inclusion may be reconsidered.

53 Mr Roxburgh's evidence concludes that *"The Inovo and PDP reports put forward technically viable stormwater treatment options, which meet District Plan requirements."* He further concludes that *"The "consentability" of the proposed stormwater solutions is a greater hurdle to overcome than the technical feasibility of the stormwater management approaches put forward."* As noted in my evidence, the stormwater concept has been carefully considered and developed so as to actively avoid the risk of an active groundwater take and therefore any need for a water take consent associated with stormwater infrastructure. There are therefore no "consentability" concerns with respect to the proposed concept.

54 **Attachment 2** indicates potential basin locations which would have sufficient capacity to achieve stormwater neutrality on the site up to the 2% AEP (50-year ARI). At this plan change stage, this information is developed and provided to show that the management of stormwater to the required standards is achievable. As noted above however, these concepts have been developed using very conservative principles for both the volume sizing of the basins and the assumptions that groundwater level will be near surface. These basin locations or sizes are not fixed, and their final form are subject to detailed design. If more area is needed to achieve the design requirements, this is what will be required and mandated by the subdivision consent process.

Wastewater Infrastructure

55 In Paragraph 27 of his evidence, Mr Roxburgh states that he does not agree with the capacity assessment set out in the Inovo infrastructure report and concludes there is no spare capacity in the existing rising main. I disagree with this statement and have set out in Paragraphs 37 to 44 of my evidence above why I conclude there is capacity within the pressure main for a minimum of 250 additional lots from the proposed development.

56 Mr Roxburgh has based his conclusion on the existing number of pump starts in 2022 (average of 11.2 per day over 253 days from 22/4/22 to 31/12/22) and the operating pressure at the pump station of 77m.

57 From the Bradleys Road pump station operating data which I have received from Waimakariri DC (pump starts, flow, tank level and pressure data at 1-minute intervals from 1/7/2021 to 7/6/2023), the average daily number of pump starts at Bradleys Road Pump Station was 3.5 pump starts per day. For the period Mr Roxburgh states, the average number of pump starts in the data I received was 4.3 starts per day. This is a very low number, typically wastewater pump stations are designed so as to ensure that pump starts are no more than 10 starts per hour.

58 Mr Roxburgh also raises concern that the design operating pressure of the pump station is 77m, typically a pressure main would be rated to at least 1.5 times the operating pressure. The Bradley Road Pump Station pressure main is rated to 100m. I do note that, in the Bradleys Road pump station design report, it is stated that the *"design pump duty is 21.7 /s at a total head of between 7 and 77m".* From a brief analysis of the pressure main, a relatively high friction factor would need to be applied to the pipe in order for dynamic head losses to require a total head of 77 m. The pumps used at the pump station are progressive cavity pumps which are designed to provide a relatively narrow range of flow over a large range of operating heads. I would interpret the above statement in the design report as saying that the design pump duty flow is 21.7 L/s which can be supplied by the pumps at a total head of between 7 m and 77 m. As opposed to inferring that the duty operating point is 21.7 L/s at a head of 77 m as Mr Roxburgh appears to have done.

59 I note from the data I received from Waimakariri DC that the maximum pressure at the pump station while pumps were operating was approximately 490 Kpa or 49 m. There were two days, 10/9/2021 and 21/4/2022 when the pressure at the pump station briefly spiked to over 100m. Outside of these isolated incidents the pipeline pressure never exceeded 494 Kpa. I am aware that there is a control valve on the existing pressure main close to the WWTP which closes when pumps are off to keep the rising main as full as possible and limit the potential for large expulsions of malodorous air on pump start up. I would expect that these pressure spikes were the result of a malfunction or communication error between the pump station and this valve. There is a pressure relief valve at the pump station to protect the pipeline from overpressure in the event of such a malfunction.

60 On my review of the data I received from Waimakariri DC, I would conclude that the existing pump starts are not excessive and the pressure main operating pressures are well within the design envelope for a PN10 (100 m head) rated pressure main. I would therefore disagree with Mr Roxburgh's basis for concluding that the pressure main was operating at capacity. In the event, that the new pump station was to connect with a flow of 12.4 L/s, the operating pressure of the main while this pump station was operating would be less than the current operating pressure when the Bradleys Road pump station is operational.

Comments on Submissions

61 A number of submissions (Submissions 382 (D. Leslie), 230 (D Myall) and 73 (L Hurley & C Stephen)) focus on high groundwater and the challenges of providing stormwater attenuation and detention at the site. As outlined in my evidence, I am of the view

that appropriate stormwater can be provided to meet the requirements of the WDC Code of Practice and ensure that stormwater neutrality is achieved within the development.

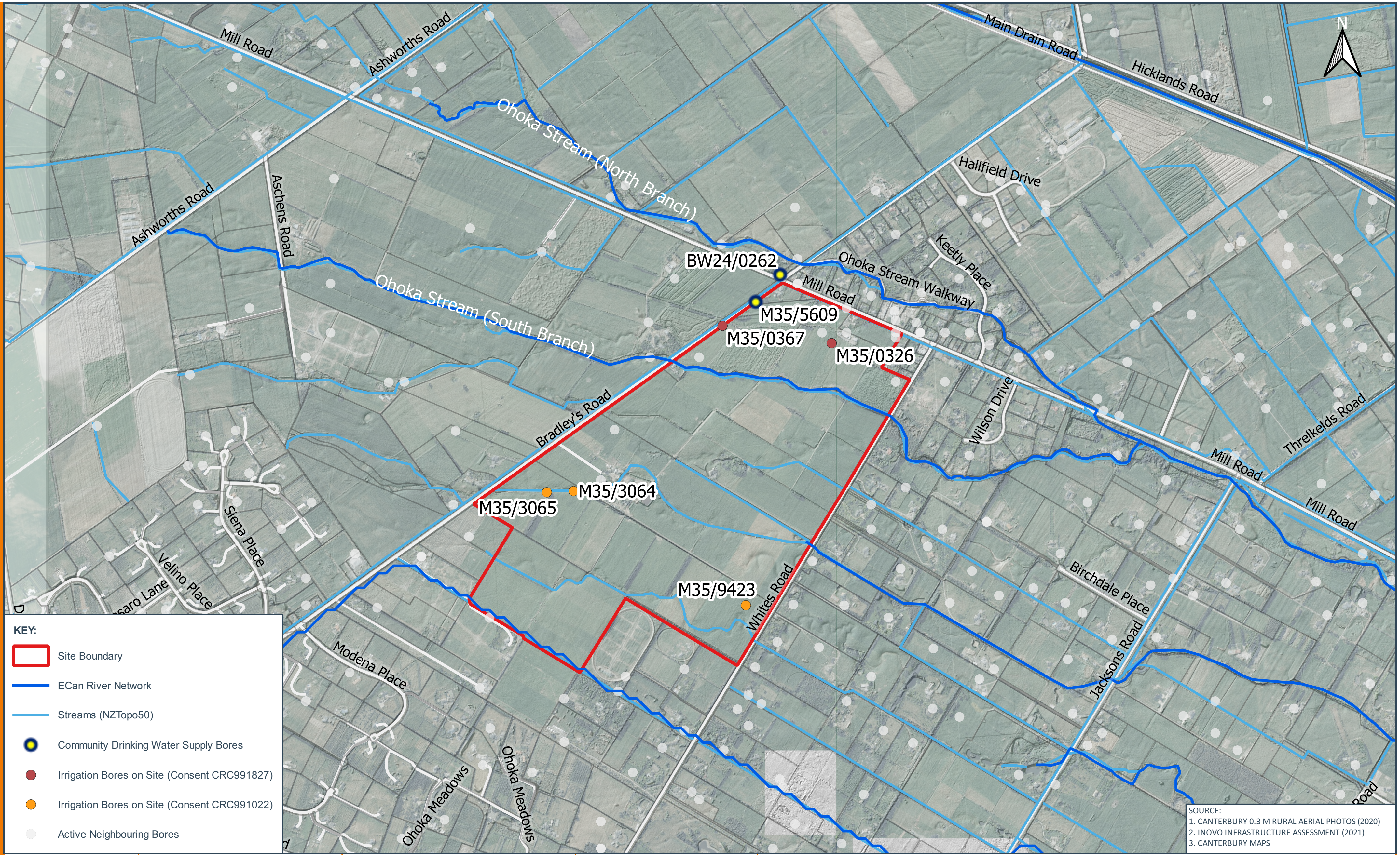
CONCLUSION

62 As outlined above, I am of the view that viable stormwater and wastewater concepts exist for the servicing of the proposed plan change area.

Dated: 6 July 2023

Eoghan O'Neill

ATTACHMENT 1



pdp

0 200 400 600 m

METRES

SCALE : 1:20,000 (A4)

NO.	REVISION	DATE	BY
A	FINAL	JUN 23	TG

CLIENT

ROLLESTON INDUSTRIAL DEVELOPMENTS LTD

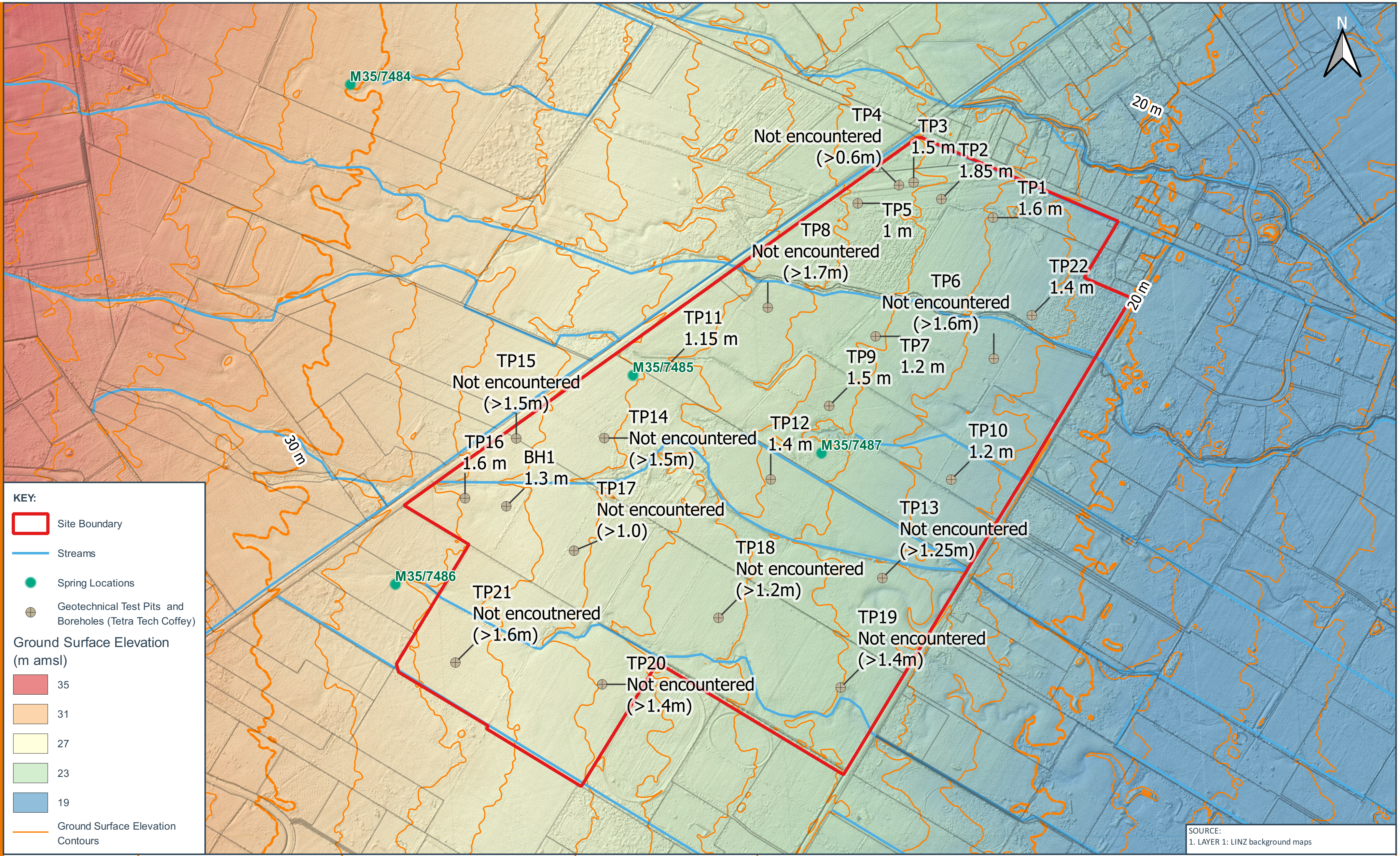
FIGURE

FIGURE 2: SITE OVERVIEW

PROJECT

PRELIMINARY WATER SUPPLY ASSESSMENT - OHOKA PLAN CHANGE

ATTACHMENT 2



0 100 200 m

METRES

SCALE : 1:12,000 (A4)

NO.	REVISION	DATE	BY
A	FINAL	JAN 22	TG

CLIENT

ROLLESTON INDUSTRIAL DEVELOPMENTS LTD

FIGURE

ATTACHMENT 2: SITE TOPOGRAPHY & GROUNDWATER LEVELS

PROJECT

HYDROLOGICAL ASSESSMENT - OHOKA PLAN CHANGE